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ABSTRACT

In the practical seismic analysis for the project of South Park Bridge Replacement, the caisson foundations are modeled as Winkler springs in the global bridge model. The global bridge model is then excited by depth-varying ground motions acting along the height of the caisson. The foundation model would entail Winkler springs distributed over the surfaces of caisson to represent the soil continuum underlying the foundation and passive soil pressure acting on the sides. The soil springs are nonlinear for consideration of yielding of localized soil. In addition, gapping elements can also be implemented in series with the soil springs to engage a full contact between the soil and the caisson during compression and to allow separation under tension. To establish correct Winkler soil springs, pushover analyses of the caisson on continuum soils considering the nonlinearity of the soil and the interface between the caisson and soil are performed using 3D finite element method (FEM). The solutions obtained from FEM would represent the overall deformation behavior of the caisson, and also address the stress-strain behavior of the local soil elements. The depth-varying ground motions acting along the height of the caisson are obtained by 2D site response analysis.

INTRODUCTION

The South Park Bridge crosses the Duwamish Waterway near the southern limits of the City of Seattle. The bridge carries traffic from 16th Avenue south on the north side to 14th Avenue south at the south bridge terminus. Industrial, commercial and residential properties lie close to the bridge. The existing South Park Bridge has been deteriorated significantly in recent years and is being considered for replacement. The project site and vicinity is shown in Fig. 1. Two foundation types were initially considered for the bascule piers of the new bridge; drilled shaft foundation or sunken caisson foundation. The caisson foundation type was ultimately selected. The bottoms of the caissons are expected to be at elevations -105 ft for the north caisson and -70 ft for the south caisson. The cross sections are to be 58 ft by 58 ft for both caissons (Fig. 2 and Fig. 3). In this paper, only the analyses for south caisson are included due to paper length limitation. However, the analysis method is similar for the north caisson.

A number of soil borings were drilled by Shannon & Wilson Inc. for the new bridge. The soil profile for the south caisson was based on the boring log of SB-05. Bedrock was not encountered in borings of SB-05 to a boring depth of 100 ft. According to the geotechnical report (PBAI 2007), the depth to bedrock at the site is expected between 164 ft and 328 ft.

According to the boring logs of SB-05, and referring to the generalized sub-surface profile in geotechnical report (PBAI 2007), the idealized soil layers and basic design parameters for site responses are shown in Fig. 4 for the south caisson.

SITE RESPONSE ANALYSIS

The project is located in a moderately active tectonic province that has been subjected to numerous earthquake of low to moderate strength and occasionally to strong shock during the brief 165-year record history in the Pacific Northwest. Seismicity in the region is attributed primarily to the interaction between Pacific, Juan de Fuca, and North American plates.

Acceleration Response Spectrum Curves

Shannon & Wilson Inc. performed seismic hazard studies, and recommended that the reference ground motion for the "no collapse" seismic design of the bridge be based on a 975-year return period. A functional level reference motion was also recommended based on 108-year return period. The 975-year and 108-year return period spectra were developed for Site Class E. The horizontal and vertical reference ground motions developed by Shannon & Wilson Inc. are presented in Fig. 5. Based on the subsurface conditions at the project site, the reference ground motion for Site Class E is interpreted to represent the shaking level near the ground surface.

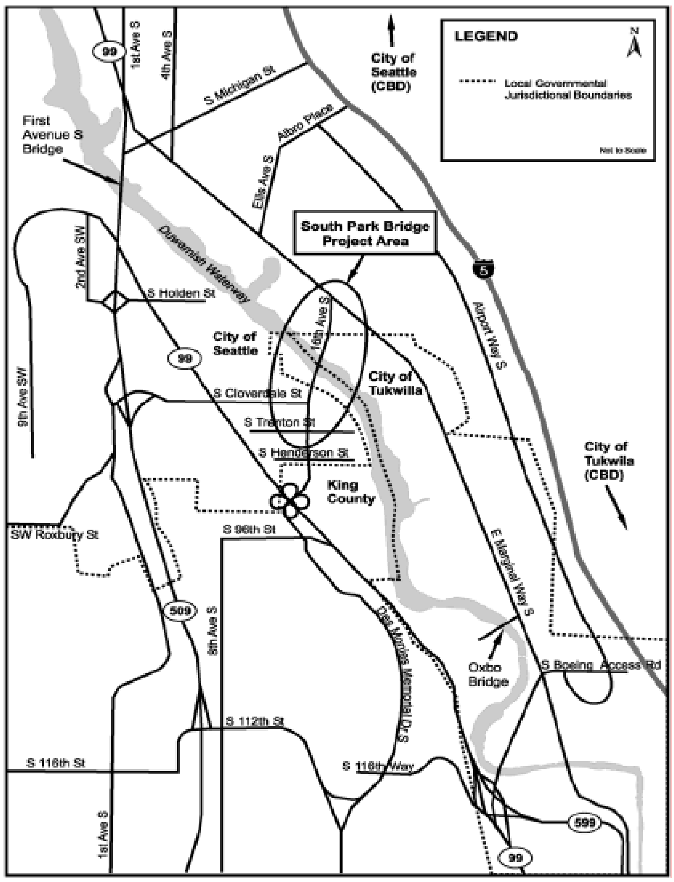


Fig. 1. Project Site and vicinity

Since the caissons supporting the main bridge are embedded relatively deep below the ground surface, the effective shaking to the bridge would come from ground shaking at some depth, probably closer to the caisson bottom. A more rigorous approach would employ depth-varying motions along the caisson height adhering to a wave propagation theory.

Generation of Time Histories

The reference ground motion for the project site was developed by Shannon & Wilson Inc. based on probabilistic seismic hazard analysis. To provide insight into what events (magnitude, distance, epsilon) are the most important for the hazard at a given ground motion level, the hazard curve can be broken down into its contributions from different earthquake scenarios. This process is called deaggregation (e.g. Bazzurro and Cornell 1999). We conducted the deaggregation analysis to assist selection of appropriate acceleration time histories for spectrum matching.

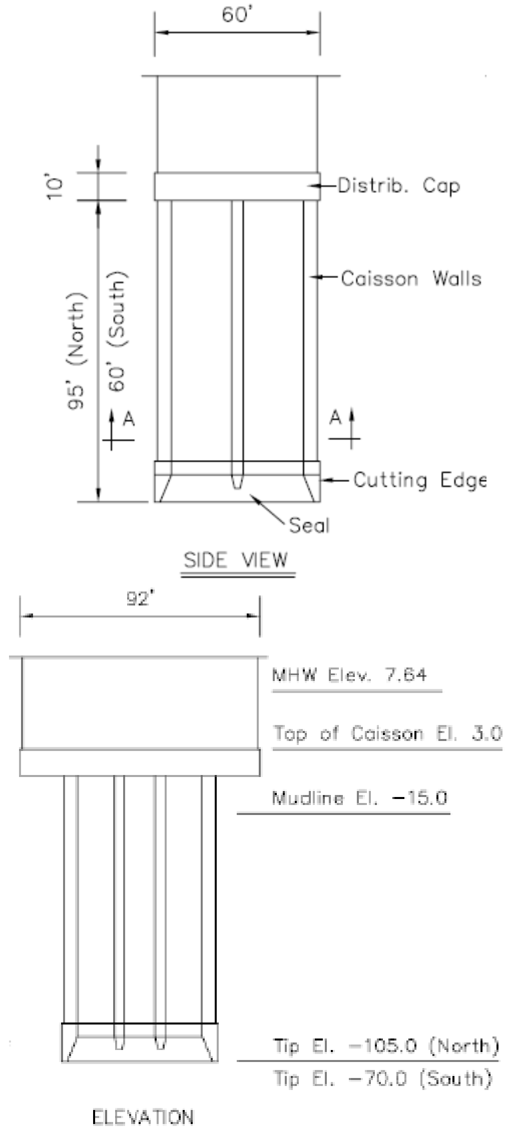


Fig. 2. Caisson profiles (side views and elevation)

Based on the deaggregetaion analysis, the seismic hazard is dominated by M 6.0-7.5 earthquakes at distances of 10-20 km. From these scenarios, the following natural earthquake records were selected as seed motions for the 975-year spectrum matching:

Set 1: 1979 Imperial Valley Earthquake at Array 6 in Plaster City
Set 2: 1989 Loma Prieta Earthquake at Fremont - Mission

Set 3: 1976 Gazli Earthquake at Karakyr
Set 4: 1994 Northridge Earthquake at LA - Centinela St

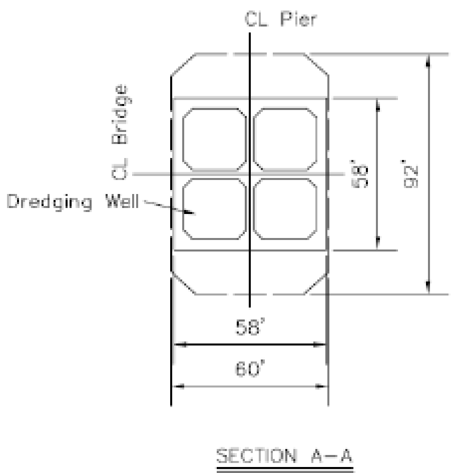


Fig. 3. Caisson profile (plain view)

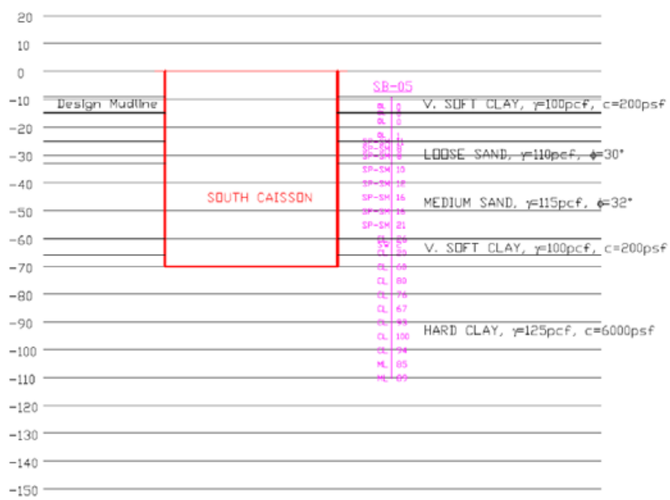


Fig. 4. Soil profile for south caisson

In addition to meeting the magnitude and distance criteria, considerations were given such that the selected seed motions should have a spectral shape that closely matches the reference spectrum. For this, we employed a computer program to search the seed motions. An example acceleration spectrum of the horizontal components of these four seed motions had been adjusted to match the design acceleration spectra as shown in Fig. 6, taking 975-year Motion Set 1, fault normal (FN) component, as a demonstration. The FN component was used for the bridge longitudinal direction and the fault parallel (FP) component was for the bridge transverse direction. Only horizontal components were used in wave scattering (site response) analysis, while the vertical component was used directly in the bridge structure analysis.

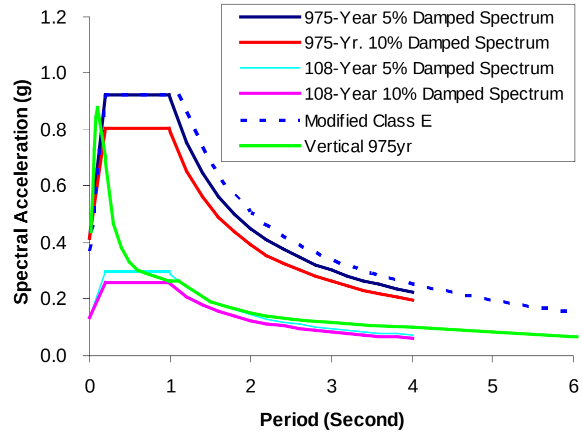


Fig. 5. Design spectral acceleration

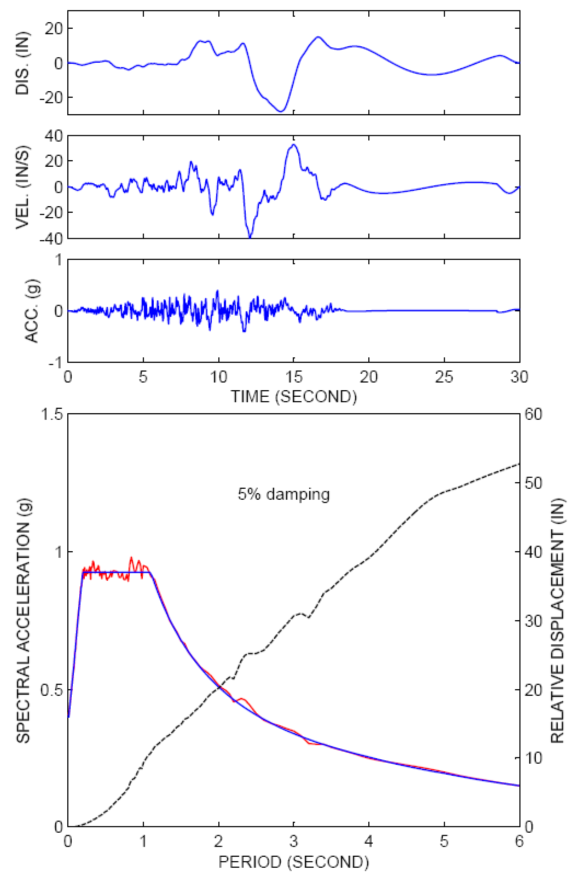


Fig. 6. 975-year Motion Set 1, FN

<u>Shear Wav Velocity Estimation</u>

The small strain shear modulus for sandy soils was estimated by the equation (Seed et al. 1984):

$$G_{\max} = 20\left(N_{1,60}\right)^{\frac{1}{3}}\left(\sigma'_m\right)^{\frac{1}{2}} \tag{1}$$

where $G_{\max}$ and σ'_m are small strain shear modulus and effective mean overburden pressure in psf, and $N_{1,60}$ is the normalized SPT blow-counts. The small strain shear wave velocity for clayey soils was estimated by the equation (JRA2002):

$$V_s = 100\left(N_{1,60}\right)^{\frac{1}{3}} \tag{2}$$

where V_s is in m/s. The relation between $G_{\max}$ and V_s is

$$G_{\max} = \frac{\gamma}{g} V_s^2 \tag{3}$$

where γ is the total unit weight and g is the gravity constant.

Using the above small strain shear wave velocity and shear modulus relations, the estimated shear wave velocities and actual input shear wave velocities in SHAKE are provided in Fig. 7 for the south caisson.

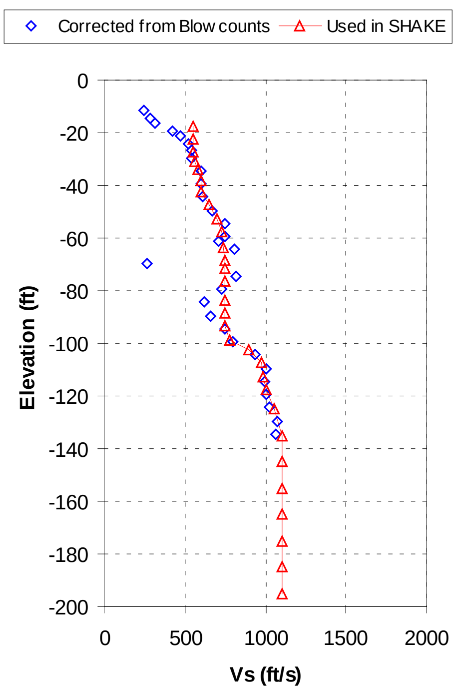


Fig. 7. Estimated and idealized shear wave velocities

1D Site Response Analysis by SHAKE

One dimensional (1D) site response analyses were conducted at the two caisson sites using the computer program SHAKE (Idriss and Sun 1992). The analyses yielded free-field soil motions at different depths for horizontally layered soils without the caisson. If the caisson is present, the free-field motions as computed by SHAKE will be altered. Such affects, known as wave scattering, are addressed by two-dimensional (2D) site response analyses using the program SASSI described in the next section. Nonetheless, 1D site response analyses serve as a benchmark case for comparison with 2D site response analysis.

The reference ground motion time history was prescribed at the ground surface consistent with the site Class E assumption. Free-field motions were computed at different depths. Strain-dependent shear modulus and damping ratios used in this analysis were in accordance with the relations by Vucetic and Dobry (1991), representing clay materials having various plasticity index values and in accordance with the relations recommended by Sun et al. (1988), representing sandy materials under various confining in-situ stress levels. Through iteration, the final linearized values of shear-modulus and damping ratio were compatible with soil shear strains equal to 65 percent of their maximum values.

2D Site Response Analysis by SASSI

Inclusion of large caisson in soil tends to alter the free-field ground motions due to relatively large wave lengths implied by the caisson dimensions. To capture such wave scattering phenomena, 2D site response analyses were performed using the computer program SASSI (Lysmer et al. 1999).

The soil was modeled as viscoelastic horizontal layers on a semi-infinite viscoelastic halfspace. The SASSI site model was constructed from the secant (strain-compatible) soil properties derived from the 1D free-field site response analysis from SHAKE.

The SASSI models used in the analysis were showed in Fig. 8. For the south caisson, the structure was modeled by 192 2D solid elements. Mass was not considered for the caisson model, which instead is left to structural analysis. Thus, this caisson model merely served as kinematic constraints to the soil nodes around the caisson in the 2D site response analysis. However, inertia interaction of the caisson was not part of the SASSI analysis due to the massless caisson. The resultant ground motions from this 2D site response analysis become kinematic motions. Since the mass of the caisson should be included in the global bridge model, the inertia interaction of the caisson will ultimately be considered by the structure engineers.

The input reference motion was prescribed at ground surface (as free-field motion) in the SASSI analysis, and acceleration time histories were computed at specified nodes around the caisson. The computed acceleration time histories were double integrated to yield displacement time histories which were electronically transmitted to the structural designer. Only horizontal motions were derived from the 2D site response analysis, while vertical motion was based on the reference motion without any site response analysis which is the current state of practice.

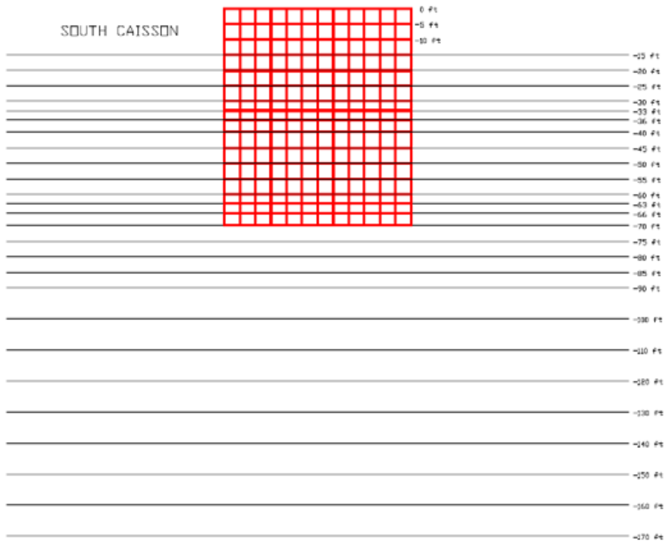


Fig. 8. SASSI mesh and soil layers for south caisson

To appreciate the shaking levels implied by the outcomes of the 2D site response analysis, response spectra of acceleration time histories computed from SASSI are presented in Fig. 9. For a comparison purpose, the results of 1D site response analysis from SHAKE are also plotted. As expected, the shaking levels reduce with depths. Although SHAKE results suggest substantial variations of shaking levels at different depths, SASSI results show much more uniform shaking levels along the caisson height due to the kinematic constraint provided by the stiffness of the caisson.

FOUNDATION MODEL

To establish correct Winkler soil springs for use in the global bridge system, pushover analyses of the caisson on continuum soils were performed using a finite element method. The solutions obtained from the finite element analysis would represent the overall deformation behavior of the caisson, and also address the stress-strain behavior of the local soil elements. The Winkler springs were extracted from the pushover analyses that would have captured the geometric non- linearity due to foundation uplift and the ultimate limit state of the foundation.

Pushover Analysis

The finite element program, ADINA (2001), was used in the soil spring analysis. The soil was modeled as Drucker-Prager material, in which two important input material constants α and κ can be calculated from the soil internal friction angle ϕ and the cohesion C:

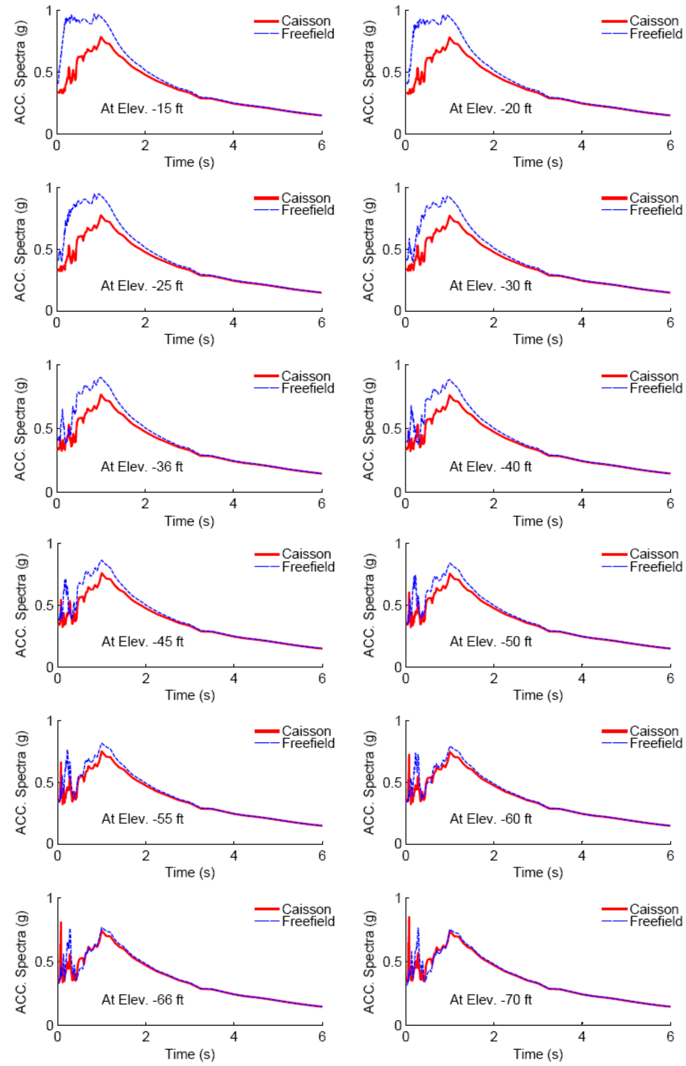


Fig. 9. Acceleration spectrum at various depths, 975 year motion 1

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)};\ k = \frac{2C\cos\phi}{\sqrt{3}(3-\sin\phi)} \qquad (4)$$

The Drucker-Prager yield function is given by

$$f = \alpha I_1 + \sqrt{J_2} - k \qquad (5)$$

where I1 is the first stress invariant, and J2 is the second deviatoric stress invariant. The Drucker-Prager model is assumed elastic perfectly-plastic.

The caisson was modeled as linear elastic material with the same Young's modulus and Poisson's ratio as a typical concrete (Table 1). Contact elements with Coulomb-type friction coefficient equivalent to a sliding angle of 25º were used to model the caisson-soil interfaces. This contact element can capture the effects of separation and friction between the caisson and the soil. The summary of the material constants is tabulated in Table 1.

Table 1. Material Constants for the FEM model

	Sand	Clay	Caisson	Interface
Young's Modulus (ksf)	576	3,400	720,000	
Poisson's ratio	0.35	0.45	0.20	
Friction Angle	32º	0º		25º
Unit Weight (pcf)	110	125		
Cohesion (psf)	0	6,000		

Due to the symmetry, only a half configuration was utilized for the analysis. The finite element mesh was divided into eight-node solid brick elements (Fig. 10) with four pairs of contact surfaces. The results are interpreted as full caisson (already multiplied with a factor a two to account for one-half model).

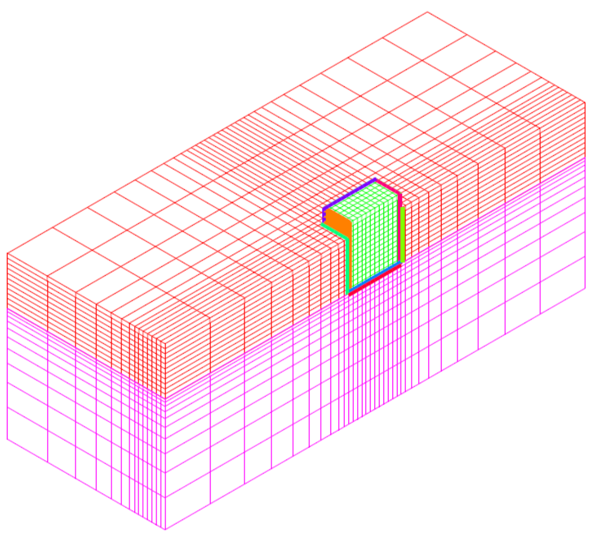


Fig. 10. FEM Mesh for South Caisson

After the finite element model was established, pushover analysis was conducted involving the following steps:

<u>Step 1</u>. Self-weight analysis: This is to set the initial state of stress in the soil elements, to set the initial normal stress in the interfaces, to obtain the caisson settlement caused by the self-weight. If the caisson is below the water, the self-weight of the caisson is then the buoyant weight.

<u>Step 2</u>. Vertical loading analysis: The caisson is vertically loaded at a specific node (e.g. at the gravity center) downward with a displacement control strategy. The self-weight load is also included. The vertical displacement-load curve is thus obtained by subtracting the settlement caused by the pure self-weight in Step 1. The uniformly distributed spring values at the base will be obtained by dividing the load with the caisson base area.

<u>Step 3</u>. Lateral pushover analysis: The caisson is first loaded with self-weigh and the design vertical dead load with a load control strategy. The caisson is then laterally pushed at a specific node (e.g. at the gravity center). A lateral displacement versus load curve will be given by this lateral pushover analysis.

Figure 11 shows the vertical load versus settlement relation from the above vertical loading analysis with or without side friction. Figure 13 presents the relationship of horizontal load versus horizontal displacement taken at the center of gravity from the lateral pushover analysis. Figure 14 is the plots of moment (applied horizontal load multiplied by the height of the center of gravity) versus the caisson rotation.

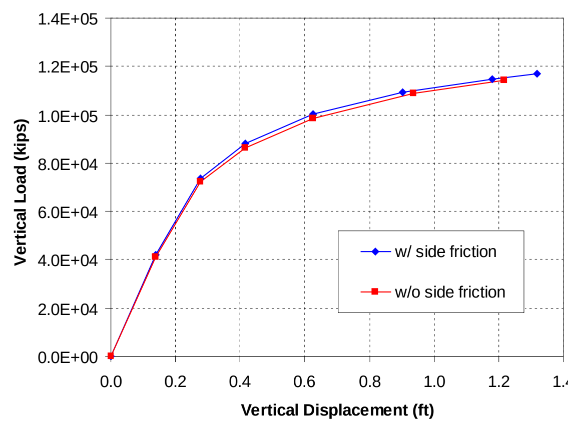


Fig. 11. Displacement and Load Relations for South Caisson

<u>Development of Winkler Springs</u>

Separate pushover analysis was conducted for the caisson supported on Winkler springs. The characteristics of the Winkler springs were established by matching the overall behavior of the pushover solutions from the continuum model. While much of the efforts were guided by the load and displacement registered in the contact elements during the continuum pushover analysis, as well as by the principle of soil mechanics, some degrees of trial-and-error were also involved. Four types of Winkler springs were developed (Fig. 16):

1. Base contact (vertical) springs
2. Base friction (horizontal) springs
3. Side friction (vertical) springs
4. Side passive pressure (horizontal) springs

The schematics of the base contact springs and side passive pressure springs are shown in Fig. 12.

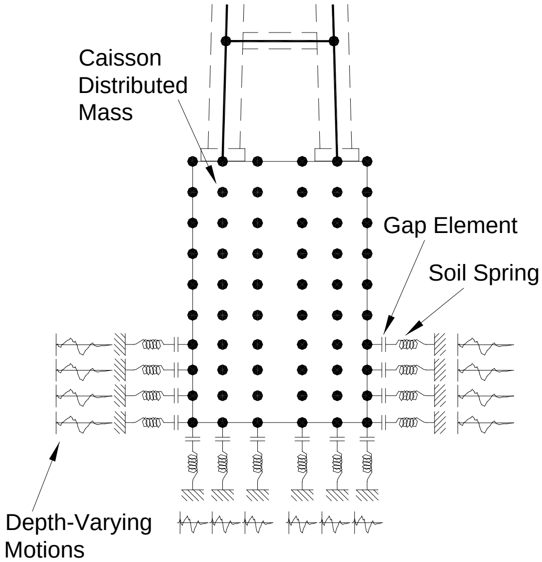


Fig. 12. Schematics of base contact springs and side passive pressure springs

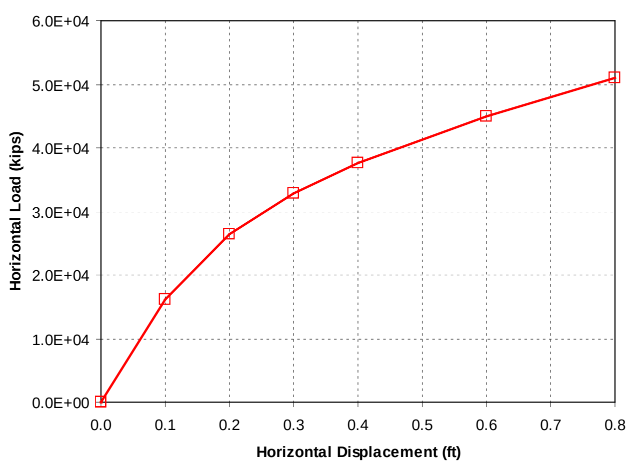


Fig. 13. Displacement and Load Relations for South Caisson

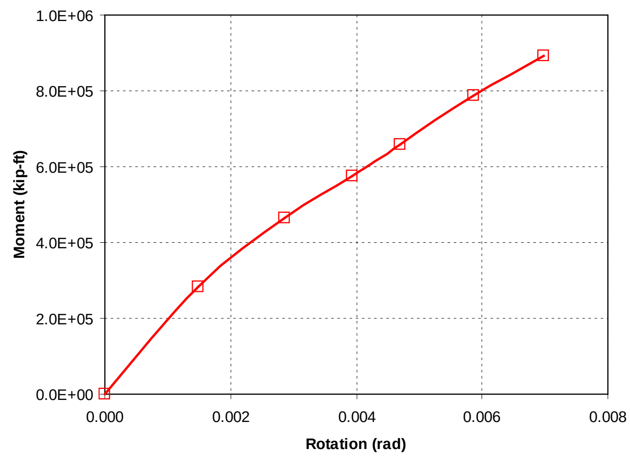
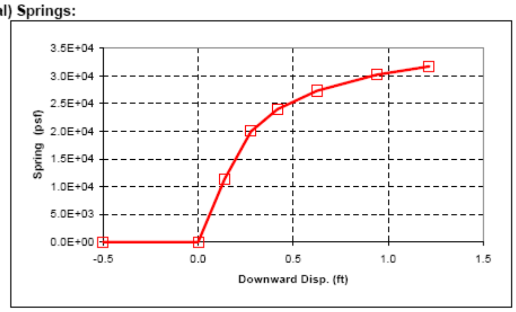


Fig. 14. Rotation and Moment for South Caisson

Coordinates of these springs are provided for the unit area basis. Therefore to implement in the structure model, discrete soil springs can be developed by multiplying with the tributary area. The pushover analysis results using Winkler springs models are compared with the pushover results of the continuum model in Fig. 17.

Base Contact (Vertical) Springs:

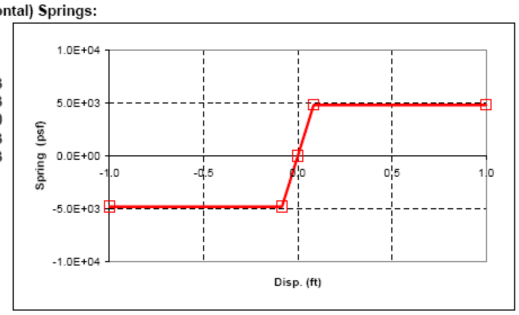
Disp ft	Spring psf
-0.500	0.00E+00
0.000	0.00E+00
0.139	1.14E+04
0.278	2.00E+04
0.417	2.40E+04
0.625	2.73E+04
0.938	3.02E+04
1.216	3.17E+04



Base vertical springs are assumed uniformly distributed at the caisson base

Base Friction (Horizontal) Springs:

Disp ft	Spring psf
-1.000	-4.80E+03
-0.084	-4.80E+03
0.000	0.00E+00
0.084	4.80E+03
1.000	4.80E+03



Base friction springs are assumed uniformly distributed at the caisson base

Fig. 15. Base Springs for South Caisson

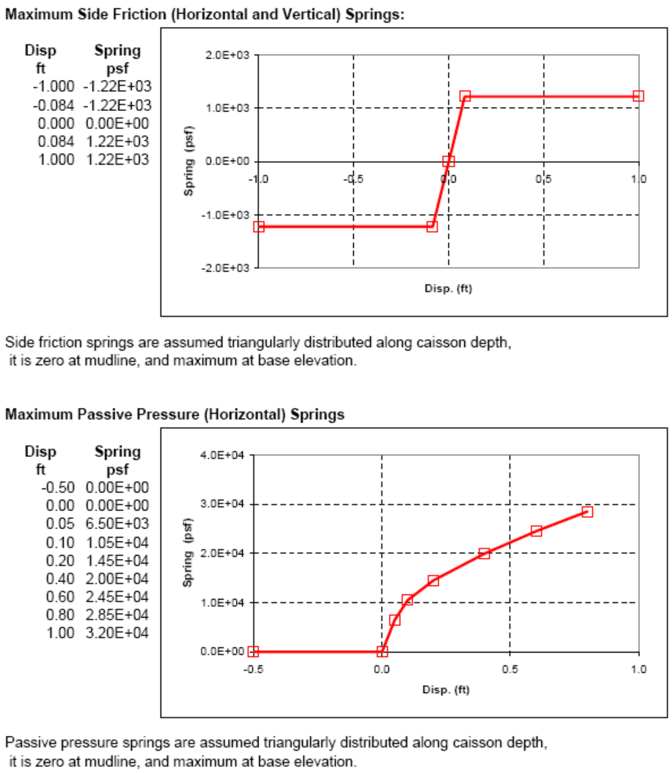


Fig. 16: Side Springs for South Caisson

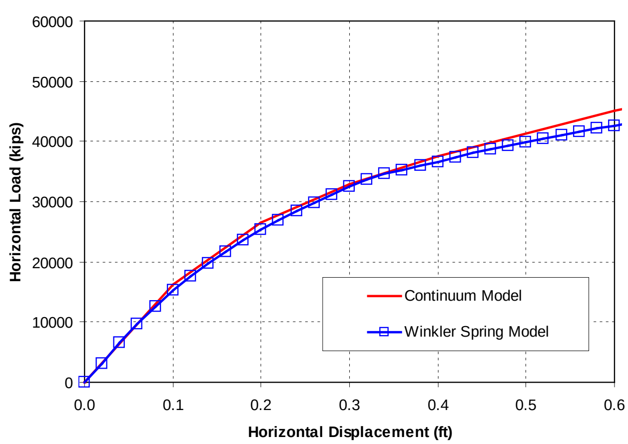


Fig. 17. Horizontal Displacement and Load Comparison of Continuum Model and in Winkler Spring Model

Base Displacements and Pressure Time Histories

With the developed Winkler springs and the depth-varying motions of the caisson, the caisson base displacement and pressure time histories can be obtained from global structure dynamic analysis for all motions. Figure 18 demonstrates the base displacements time histories at the four corners and the center of the caisson base for 975-year Motion 1 (M1). Figure 19 presents the corresponding approximate pressure time histories.

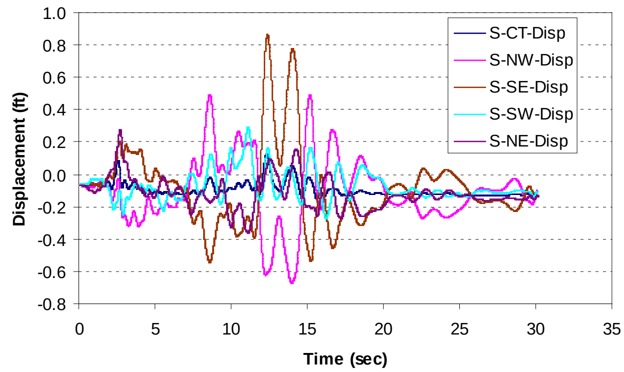


Fig. 18. Displacement time histories at bottom of South Caisson of 975-year Motion 1

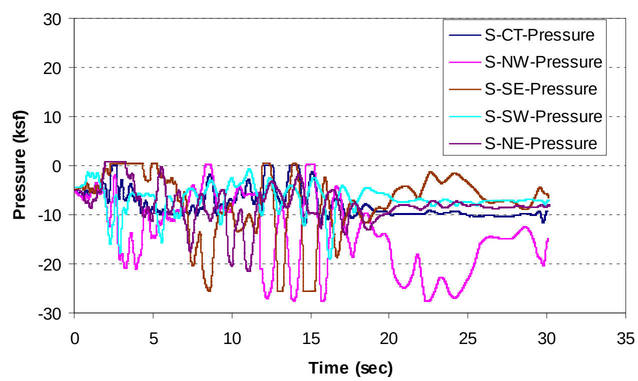


Fig. 19. Soil pressure time histories at bottom of South Caisson of 975-year Motion 1

Base Pressure

In order to check the compatibility of the calculated base pressures and displacements obtained from the global bridge model, two cases were analyzed for the separated caisson model in continuum soils: (a) at maximum rotations with associated vertical displacements of center point; (b) at maximum veridical displacements of center point with associated rotation.

Based on the displacements (four corners and center point) time histories at the bottom of caissons for earthquake motion 1, 2 and 3 (M1. M2 and M3), we summarize the maximum rotations with associated vertical displacements of center point and maximum veridical displacements of center point with associated rotation (Table 1). The maximum rotation is estimated by the maximum difference vertical displacements of any two neighboring corners divided by the distance of these two corners. When the associated vertical displacement of center point at maximum rotation is positive, more critical value of zero will be input for FEM analysis. The same mesh, material parameters, and methods as those to develop Winkler springs are used here.

The FEM analysis procedure is as followed:
(1) Apply gravity load and dead load of the caisson
(2) Apply the imposing vertical settlement
(3) Apply horizontal displacement at the gravity center consistent to the rotation angle (the horizontal displacement divided by the gravity center high over the base equals to the rotation angle)

Presented in Fig. 20 are caisson base contact pressure distributions (calculated by contact forces divided by the tribute area). These base pressures are consistent with the pressure time histories obtained from the global bridge model as shown in Fig. 19, which proves the validation of the caisson model.

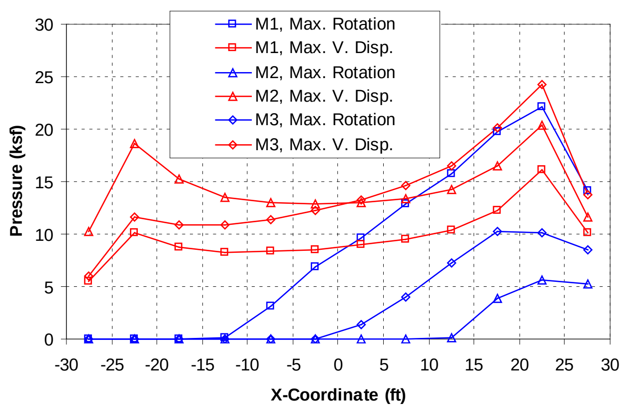


Fig. 20. Base contact pressure based on the maximum vertical displacements or rotation (South Caisson)

SUMMARY

This paper presents a practical seismic analysis for the caisson foundations which are modeled as Winkler springs in the global bridge model. The foundation model entails Winkler springs distributed over the surfaces of caisson to represent the soil continuum underlying the foundation and passive soil pressure acting on the sides. The frictional springs are also assumed on the caisson surfaces. The base contact and side passive springs are nonlinear for consideration of yielding of localized soil. In addition, gapping elements are implemented in series with the soil springs to engage separation effects. Pushover analyses of the caisson on 3D continuum soils considering the nonlinearity of the soil and the interface between the caisson and soil are performed using 3D FEM in order to establish correct Winkler soil springs. The solutions obtained from FEM represent the overall deformation behavior of the caisson, and also address the stress-strain behavior of the local soil elements. The depth-varying ground motions acting along the height of the caisson are obtained by 2D site response analysis. The developed springs and depth-varying ground motions for caissons are for the purpose of global bridge structure analysis.

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